Use of a Rammed Aggregate Pier Ground Improvement trial to improve understanding of liquefaction potential in geologically aged soils

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ABSTRACT

CMW Geosciences (CMW) with the support of CLL Service & Solutions Ltd (CLL) have carried out an in-house research and development (R&D) exercise involving a Rammed Aggregate Pier (RAP) ground improvement trial within Pleistocene-aged sand deposits that have been assessed using conventional methods to be susceptible to liquefaction. Although the results of the trial demonstrated that the Pleistocene-aged alluvial sands densified following RAP installation, shear wave velocity measurements following RAP installation were much lower than those undertaken prior to installation indicating that the high energy vibratory hammer used to install the piers had disturbed the soil microstructure. The results of our trial have highlighted an interesting effect of dynamic ground improvement techniques in geologically older soil deposits.

The updated MBIE guidelines in Earthquake Geotechnical Engineering Practice released in November 2021 have acknowledged that although the age of a deposit is an important factor to consider when undertaking a liquefaction assessment, it is often challenging to quantify the effects ageing has on the liquefaction resistance of soils with no widely accepted methods to do so. As a result, the guidelines have recognised the limitations of the methods and have outlined an approach for evaluating the ageing effects incorporating the use of shear wave velocity measurements and advanced lab testing. This provides the New Zealand geotechnical industry further guidance when assessing liquefaction potential on geologically aged soils.

Keywords: Rammed Aggregate Pier, liquefaction, ground improvement, aged soils

1 INTRODUCTION

A Rammed Aggregate Pier (RAP) ground improvement trial was undertaken by CMW Geosciences (CMW) with the support of CLL Service & Solutions Ltd (CLL) to determine if RAPs were a suitable ground improvement option to mitigation liquefaction for a significant infrastructure project located within the Bay of Plenty region. The design of high importance infrastructure in the Bay of Plenty region attracts high seismic demands, that in turn can generate significant liquefaction induced settlements and lateral displacements beneath structures where ground improvement is typically required to achieve settlement and displacement limits.

RAP ground improvements have been used for liquefaction mitigation on projects around the world. Using these projects together with recent RAP ground improvement trials conducted around the world as guidance, three different triangular arrays, representing three different area replacement ratios were constructed to assist in determining the most appropriate arrangement. Considering the results of other trials, conventional large strain Cone Penetrometer Testing (CPT) in conjunction with small strain shear wave velocity testing methods, using the seismic cone penetrometer (sCPT), were carried out prior to and following RAP installation to assist in evaluating the effectiveness of the ground improvement.

2 GEOLOGICAL SETTING

The RAP ground improvement trial was situated over elevated terrace topography at the foothills of the Tauranga basin. The terraces in this region are typically mantled by 2m to 3m of late quaternary volcanic ash comprising silty sand, sands and sandy silts overlying Pleistocene alluvium differentiated based on CPT trace characteristics. The upper 2m to 3m of the Pleistocene alluvium directly beneath the volcanic ash comprises sensitive silts followed by pumiceous sands with silt lenses that progressively increase in density with depth. The groundwater table was encountered approximately 4.0m to 5.5m below existing ground level.

The Pleistocene-aged volcanic derived pumiceous sands encountered below the groundwater table, have been assessed using conventional methods to be susceptible to liquefaction.

3 RAMMED AGGREGATE PIER TRAIL

The RAPs were installed by CLL using the Geopier Impact Pier System and comprised 600mm diameter aggregate columns installed through a pre-driven 250mm diameter mandrel using a proprietary impact action to densify the columns and dilate them into the surrounding soil, as shown in Figure 1a. The aggregate comprised clean screen 40/20 gravel from a local quarry with a particle size ranging from 20mm to 40mm.

Three RAP arrangements were installed, with the piers spaced at 1.25m, 1.5m and 1.8m triangular centres to represent area replacement ratios (ARR) of 20%, 15% and 10% respectively. The piers were installed to depths of approximately 15m below the ground surface, the limit of the machinery used.

Both conventional large strain Cone Penetrometer Testing (CPT) and small strain shear wave velocity testing methods, using the seismic cone penetrometer (sCPT), were carried out prior to and following RAP installation. Prior to installation a CPT in combination with a sCPT was undertaken at each of the arrays, targeting the location of one of the priers.

RAP installation was completed in early June 2021, with the post installation CPTs and sCPTs completed early August 2021. A series of three CPTs in combination with sCPTs were carried out at each array following installation, one in the centre of the array, one adjacent to the pier and one at equal distance between the centre and the edge of the pier, as shown in Figure 1b.

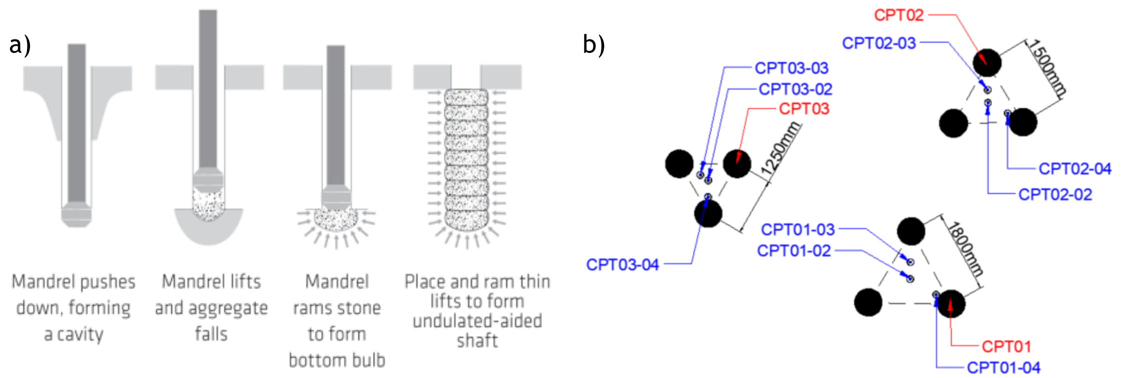

Figure 1: a) RAP construction methodology (Wissmann et al. 2015) b) Schematic of pre (red) and post (blue) installation CPT/sCPTs relative to the RAP arrays

4 POST RAP INSTALL EVALUATION

4.1 Performance Assessment

RAP ground improvements have been used for liquefaction mitigation on projects around the world, with various studies demonstrating their effectiveness in mitigating liquefaction in clean sands and more recently silty sands (Wissmann et al. 2015 & Smith and Wissmann, 2018). The effectiveness of the RAP ground improvement in various soil types is generally assessed by comparing pre and post installation CPT data such as cone tip resistance and corresponding soil behaviour type index.

The effectiveness of the RAP ground improvement is often assessed over time with CPTs and sCPTs completed at different time intervals following RAP installation. Studies like that of Saftner et al. (2018) have demonstrated time-dependent strength gain occurs with a further increase in CPT tip resistance observed one month following RAP installation and the initial post installation CPT.

Other methods incorporating shear wave velocity data have also been used to assess the effectiveness of the RAP ground improvement (Wissmann et al. 2015), with the view that the small strain stiffness (reflected in the shear wave velocity) may increase as a result of the ground improvement. Given the geological setting (ie. age and composition), seismic CPTs were also considered more appropriate with researchers noting that conventional CPT based liquefaction analysis methods can over-estimate liquefaction potential in geologically older soils and pumiceous soils (Orense & Pender, 2013).

It is acknowledged that laboratory testing of soil samples prior and following the RAP ground improvement would have also assisted in assessing the performance of the ground improvement (Orense et. al 2020). However due to constraints, soil samples were unable to be retrieved.

4.2 Standard Cone Penetration Data

4.2.1 Cone Tip Resistance

The Pleistocene-aged alluvial sands considered to be susceptible to liquefaction were typically encountered 6m to 8m below the ground surface. The densification of these deposits was assessed by comparing the uncorrected cone tip resistance (qc) values pre and post installation. Generally, the data shows that the RAP's have resulted in an increase in qc in these soil layers, with increase in qc more significant in the arrays with higher area replacement ratios, as shown in Figure 2.

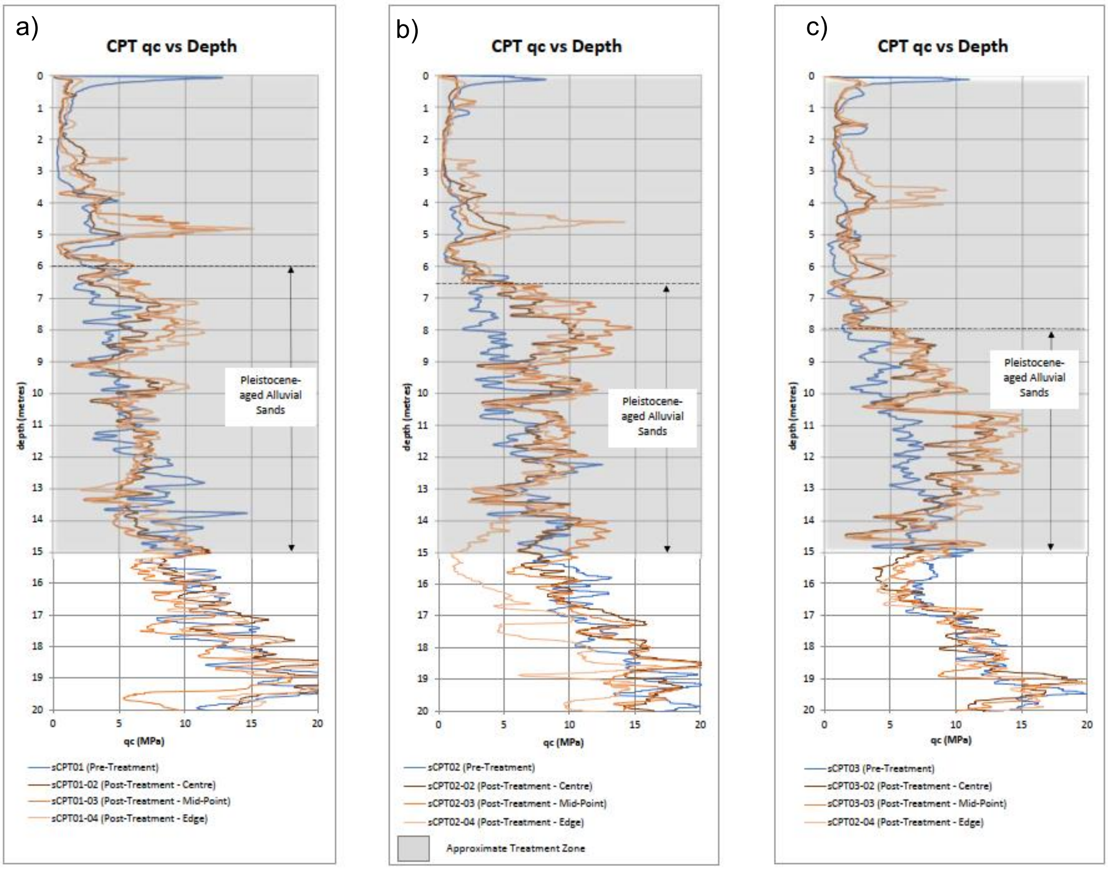

Figure 2: Pre and post installation CPT qc vs depth a)1.8m spacing (ARR = 10%), b) 1.5m spacing (ARR = 15%), c) 1.25m spacing (ARR = 20%)

4.2.2 Improvement Ratio

Vautherin et al. (2017) suggested an improvement ratio (Qc), as a method of assessing the improvement of the soil, with the improvement ratio being compared to various influencing factors such as soil behaviour type and area replacement ratio. The improvement ratio is the ratio between the post improvement qc and the pre improvement qc. Typically for studies with large data sets this is done for each select layer, in this case Qc was determined for the full depth of the trace at 1cm intervals.

A comparison of the improvement ratio as a function of pre improvement qc with respect to the Robertson (1990) soil behaviour type index, Ic, was carried out for each array. The CPT undertaken in the centre of each array was used for the post improvement qc, as this demonstrates the most conservative improvement ratio.

For the soils considered susceptible to liquefaction (Ic < 2.6), higher pre improvement qc showed the least improvement, with a portion of the soil profile showing a reduction in qc following RAP installation with Qc values less than 1, as shown in Figure 3.

The overall trend of higher pre improvement qc showing the least improvement is similar to Vautherin et al. (2017) study which was undertake in Christchurch following the 2010 and 2011 earthquake (ie. geological young soils). However, Vautherin et al. (2017) generally observed improvement following RAP installation with the Qc greater than 1.

ARR of 20% showed the most improvement for the soils with high pre improvement qc. This is consistent with Vautherin et al. (2017) with higher ARR showing greater improvement. Although it is noted that their study had a limited data set for ARR greater or less than 8%.

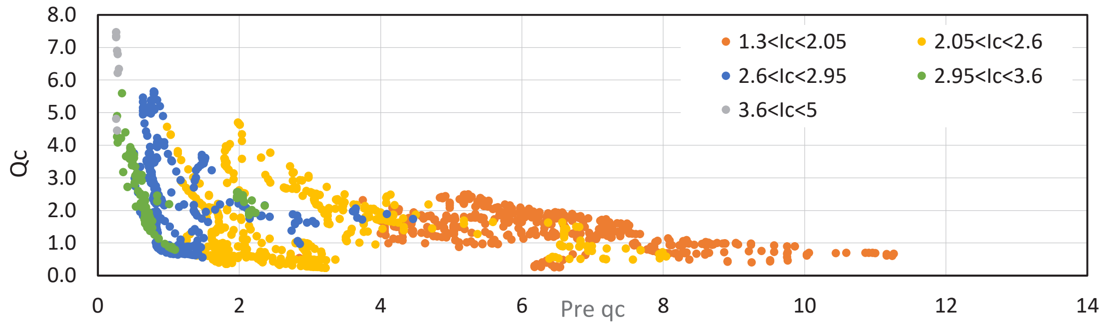

Figure 3: Improvement ratio (Qc) as a function of pre qc for ARR = 20%

4.3 Seismic Cone Penetrometer Data

4.3.1 Shear Wave Velocity

Comparison of the small strain shear wave velocity data obtained from the sCPTs pre and post RAP installation showed no improvement to the soil layers following installation. In fact, the shear wave velocity measurements post installation were lower than those record prior to installation, with average 20% reduction in shear wave velocity across the treated soil profile.

These results were unexpected, with a similar trial (Wissmann et al. 2015) showing a slight increase in shear wave velocity following ground improvement. In light of this, the authors used the shear wave velocity data to further evaluate the effects of the ground improvement on the soil properties / parameters, as discussed in the following sections.

4.3.2 Rigidity Index

The normalised rigidity index (K_G) has been proposed by Robertson (2015) as a parameter that combines Vs and normalized CPT tip resistance (Q_{tn}), to detect and quantify the presence of microstructure. The K_G of the Pleistocene-aged alluvial sand prior to RAP installation typically ranged between 250 and 360, with a clear reduction in K_G, to between approximately 110 and 150, following RAP installation. Generally, the lower K_G were observed within the sCPT carried out at the edge of the pier.

Schneider and Moss (2011) indicated that soils with no microstructure (ie. geologically young soils with no bonding), have rigidity index values which range between 110 and 330, average 215. Considering these values, it appears the RAP ground improvement has result in soil microstructure consistent with that of a geological “younger” soil.

4.3.3 Strength Gain Factor

This observation is further supported when comparing the strength gain factors for the soil profile prior and following installation of the RAP's.

Using the MEVR (ratio of measured to estimated shear wave velocity) method proposed by Andrus and Hayati (2009) a strength gain factor of between 1.1 and 1.4 was determined for the Pleistocene-aged alluvial sands encountered at depths below 6m to 7m, prior to the installation of the RAP's. Following RAP installation this reduced to between 0.8 and 1.0

The MEVR method was compared with the one proposed by Robertson (2015) using the rigidity index. This demonstrated that the Robertson (2015) method is more conservative, with strength gain factors in the order of 1.1 to 1.4, pre treatment and in the order of 0.7 and 0.8 following installation of the RAP's.

The reduction in strength gain factors following RAP installation, also suggests that the ground improvement has change the microstructure of the Pleistocene-aged alluvial sands with a soil microstructure which now appears consistent with that of a geological "younger" soil.

4.3.4 Shear Wave Velocity Improvement Ratio

Recent studies demonstrated that shear wave velocity ratio typically increases with increase in ARR for ground improvement using columnar inclusions (Boulanger and Shao 2021 and Rahmani and Baez 2020). However, as discussed above this was not observed during the RAP ground improvement trial, with a reduction in shear wave velocity observed post treatment.

Plotting the shear wave velocity improvement ratio as a function of pre improvement shear wave velocity, showed an increase in shear wave velocity ratio with an increase in ARR. However, the shear wave velocity ratios are generally below 1 (refer to Figure 04) indicting no improvement post treatment.

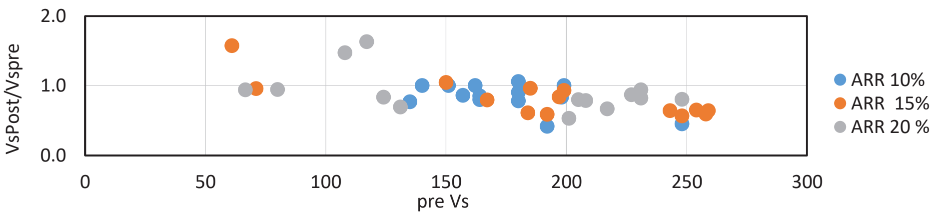

Figure 4: Shear wave velocity improvement ratio as a function of pre improvement shear wave velocity

5 DISCUSSION

The RAP ground improvement trial identified that the microstructure of the soil has been disturbed as a result of the high energy vibratory hammer associated with installing the piers. Although the RAPs had densified the soil, the soil microstructure now appears consistent with that of a geological "younger" soil. These observations are not consistent with similar ground improvement studies (Wissmann et al. 2015).

It is noted that current in-situ testing methods and subsequent analysis methods used in evaluating liquefaction potential of soils are generally based on a large database of case histories involving very young, silica-rich soils which have no bonding/cementation (Robertson, 2015). Therefore, there are limitation associated with assessing geologically aged soils as well as those with a high pumice content.

Following RAP installation, it has been acknowledged that the soils "heal" over time with verification testing typically undertaken a few months following installation (Saftner et al. 2018). Although, this has typically been observed with an increase in tip resistance, opposed to shear wave velocity. Further research into the time-dependent strength gain aspect of this trial is required, with further testing to assess whether there has been a change in the microstructure since the testing in August 2021.

Based on a number of ground improvement trial undertake in Christchurch following the 2010 and 2011 earthquakes the MBIE suggested that a ARR of 8% is reasonable to achieve the required densification to provide resistance to the effect of liquefaction. However, this RAP ground improvement trail suggests a higher ARR may more suitable. Although, this is likely to be attributed to the microstructure of the soil as well as the pumice content, it highlights the importance in understanding the soil properties when assessing liquefaction potential and associated ground improvements options.

6 CONCLUSION

Although the trail was not successful in providing an appropriate ground improvement option for the project, it raises some interesting questions about the microstructure present in geologically aged soils and how this may change over time.

This RAP ground improvement trial further supports the use of "ageing" when assessing liquefaction potential. However, as with recent studies, it demonstrates that the "age" of soils can be reset following a seismic event with the microstructure (ie. bonding / cementation) of the soil being disturbed. As such the use of empirical formulas based on the geological of the soil are not considered appropriate when assessing liquefaction potential of geologically aged soils as these give no consideration the time since the last critical disturbance.

This is consistent with the updated MBIE guidelines which have recognised the limitations of the methods used to quantify the effects ageing on the liquefaction resistance and have outlined an approach for evaluating the ageing effects incorporating the use of shear wave velocity measurements and advanced lab testing.

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